Update to UFC 3-340-02 for Blast Resistant Design of Masonry Components

by:

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Abstract

The Department of Defense Explosives Safety Board (DDESB) has funded Protection Engineering Consultants (PEC) to update the Chapter 6 sections on blast resistant design of masonry components in UFC 3-340-02, "Structures to Resist the Effects of Accidental Explosions." Under this tasking, PEC will apply data from recent research and testing to develop new and updated analysis and design procedures for masonry walls. These procedures will be specifically written to satisfy the explosives safety requirements of DoD 6055.09-STD, "DoD Ammunition and Explosives Safety Standards." Wherever possible, the new UFC criteria will be written to be consistent with the current state-of-the-practice for conventional masonry construction. As part of this tasking, PEC will identify areas where more research and testing are recommended. The revised sections will continue to apply single-degree-of-freedom (SDOF) analytical models, but design procedures will be updated, in accordance with current industry practice, to use vertical steel reinforcing bars for flexural reinforcement. Flexural response criteria also will be revised. New guidance will be added on the analysis of existing unreinforced masonry components assuming brittle flexural response and arching from axial loads. References to applicable ACI 530 and ACI 318 requirements will be added, and masonry shear strength and rotational restraint provided to masonry walls by the foundation system will be addressed.

Introduction

PEC and the UFC 3-340-02 Technical Working Group (TWG) are working with DDESB to update the blast analysis and design provisions for masonry components in UFC 3-340-02. The primary objective of this revision is to provide up-to-date design procedures that both satisfy DoD 6055.09-STD explosives safety requirements and are readily accessible by an experienced blast designer. Accordingly, the update will incorporate new example problems in the chapter appendix to facilitate understanding and proper application of masonry guidance. To minimize potential misuse, the update also will clearly define limits on the use of masonry walls for explosives safety applications. The revision will include new criteria on localized damage (i.e. spall and breach), quality assurance during construction, combined shear and tension loading, compression membrane and arching response, evaluation of support provided at the base of masonry walls by the foundation system, reinforcement of walls, and flexural response criteria. References to applicable ACI 530 and ACI 318 requirements also will be added.

PEC has submitted its 35% draft masonry revision to DDESB, and it is currently under review by TWG members. The more substantive changes proposed in this submittal are summarized in the following sections.

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Masonry Strengths

Depending upon the specific application, the updated UFC may allow the use of any masonry considered adequate for design of structural walls. A requirement will be added that all new masonry walls must be constructed with a minimum amount of vertical steel reinforcement. The revision will focus on walls as the primary type of masonry component for blast design. In so doing, it only will address unreinforced masonry and masonry reinforced with steel reinforcing bars.

The updated UFC will continue to provide information on typical properties of masonry blocks and on dynamic material strengths for masonry subject to high strain rate loadings. The masonry compression strength is based on the prism strength, f'_m, of a specimen that includes three masonry units, grout, and mortar. For new design, minimum prism strength of 2000 psi is required with grouted cells. Additionally, minimum grout strength of 3000 psi is required to enhance composite action between the reinforcing steel and masonry. If prism test data are not available, the information in Table 1 may be conservatively used. The modulus of elasticity (E_m) of a masonry component can be calculated as shown in Equation 1.

The flexural strength of masonry for blast design is based on a dynamic flexural tensile strength (i.e. modulus of rupture), f_{dt}, of 200 psi. If f'_m exceeds 2000 psi, f_{dt} can be taken as 10% of f'_m provided that it does not exceed 250 psi. These values represent the dynamic adhesion strength between the mortar and masonry at strain-rates representative of masonry response to far-range blast loading. These values for f_{dt} were developed by a trial and error procedure where they caused SDOF analyses to match measured unreinforced masonry wall response well, on the average, from a large number of high explosive tests (i.e. over 50 tests) (PDC-TR 08-07, 2008). The wall response in these tests was observed primarily in terms of damage levels due to the brittle nature of unreinforced masonry response and to the limited number of measured deflections. They are applicable for modern masonry construction (i.e. since 1960) that is laid in running-bond pattern, in good condition, and without substantial infilling of former openings.

Table 1. Default Masonry Prism Strengths

Type of Unit	**Prism Strength (f'_m)**
Hollow Units	1350 psi
Hollow Units Filled with Grout	1500 psi
Solid Units	1800 psi

$$E_m = 1000\,f'_m$$

Equation 1

The in-plane shear strength and out-of-plane (i.e. diagonal shear strength) shear strength of masonry are calculated as shown in Equation 2. The net area for out-of-plane shear loading is the solid area of the masonry plus the area of any grouted voids. The critical shear section near the support is determined in the same manner as for reinforced concrete in UFC 3-340-02. The shear strength in Equation 2 is based on ACI 530 and on blast test data where CMU walls failed in out-of-plane shear (Bazan and Oswald, 2009). In cases where the applied shear force at the critical shear section exceeds the masonry shear strength, V_m, the shear strength of the shear reinforcement can be designed to carry the excess shear force. However, the shear reinforcement

should be spaced at a maximum distance of one-half the wall thickness to ensure that the reinforcement will cross the shear failure plane. This requirement causes the use of stirrups to be unpractical for vertically spanning single wythe clay tile and CMU walls because the stirrups can only be placed in the bed joints (i.e. at 8 inch spacing) and these types of masonry are only manufactured up to 12 inches thick. Therefore, masonry walls usually must be designed so that the out-of-plane masonry shear strength exceeds the design shear force.

$$V_m = 2\sqrt{f'_m}A_n$$

Equation 2

where:

V_m = dynamic shear strength per unit width along wall resisted by masonry (lb)
f'_m = masonry prism compressive strength (psi)
A_n = net cross section area for shear (in^2)
= solid cross section (i.e. exclusive of any void area) for out-of-plane shear
= area of face shells of non-solid masonry for in-plane shear
= whole thickness in all cases for solid masonry

Shear walls can be subject to combined out-of-plane and in-plane shear. In this case, the shear capacity of the wall against in-plane and out-of-plane shear loads should satisfy Equation 3. This equation is used in ASCE (1997).

$$\left(\frac{V_{uo}}{V_{mo}+V_{so}}\right)^2 + \left(\frac{V_{ui}}{V_{mi}+V_{si}}\right)^2 \le 1.0$$

Equation 3

where:

V_{mo} = out-of-plane shear force resisted by masonry (lb)
V_{so} = out-of-plane shear force resisted by shear reinforcement (lb)
V_{mi} = in-plane shear force resisted by masonry (lb)
V_{si} = in-plane shear force resisted by shear reinforcement (lb)
V_{uo} = peak applied out-of-plane shear force (lb)
V_{ui} = peak applied in-plane shear force (lb)

The case of combined shear and axial tension load is not addressed in ACI 530 since tension is not common in conventional masonry wall design. However, this case may occur in masonry walls subject to internal explosions. The tension force can be resisted with additional reinforcement added to that required for flexure so that the shear strength of masonry is theoretically unaffected. However, there are no available test data to demonstrate this response, and the masonry shear strength in a wall subject to tension is conservatively assumed equal to zero in the updated UFC.

Direct shear stresses can occur in blast-loaded components due to early time response that is dominated by a shear response mode, rather than flexure. After this very early time response, flexural response dominates and causes diagonal shear stresses. Based on far-range blast testing

data of reinforced and unreinforced masonry subjected to no axial tension load, direct shear is not a problem in this loading realm. Therefore, the 35% draft proposes that no consideration of direct shear be required in masonry walls at scaled standoffs greater than 3 ft/lb$^{1/3}$. For cases with a lower scaled standoff, the 35% draft proposes that shear friction design be used to resist the calculated direct shear force at the base of the wall using dowel rebar that is developed across the joint between the foundation and the wall. The direct shear force at the top of masonry walls is typically significantly less than that at the bottom of the walls due to further distance from the charge. In such cases, the 35% draft proposes that the wall be assumed to have adequate direct shear strength at the top if the vertical reinforcing steel is continuous through a double bond beam at the top of the wall.

There are very limited test data on the relationship between strain-rate and masonry properties. Dynamic increase factors (DIF) and static strength increase factors (SIF) for far-range loading similar to those for reinforced concrete are considered applicable for reinforced masonry, as shown in Table 2. Otherwise, no DIF or SIF should be used for masonry design.

Table 2. Dynamic Increase Factors for Masonry Design

<table>
<tr><th>Material</th><th>Response Mode</th><th>Dynamic Strength</th></tr>
<tr><td rowspan="3">Masonry</td><td>Compression due to Flexure</td><td>1.19 f'_m</td></tr>
<tr><td>Shear</td><td>1.00 f'_m</td></tr>
<tr><td>Axial Compression</td><td>1.12 f'_m</td></tr>
<tr><td>Steel Reinforcement</td><td>Tension due to Flexure</td><td>1.17f_y(SIF)</td></tr>
<tr><td colspan="3">f'_m is masonry prism strength
f_y is minimum specified reinforcing steel yield strength
SIF is the static increase factor equal to 1.1</td></tr>
</table>

Spall, Breach, and Fragment Penetration

Spall and breach of solid masonry walls (e.g. fully grouted CMU) can be predicted using the same approach as applied to reinforced concrete in Chapter 4 of UFC 3-340-02, with the exception that the masonry wall is required to have a thickness to standoff ratio that is increased by a factor of 1.5 higher than that required for concrete. Also, the masonry prism compression strength, f'_m, should be used in place of the concrete compression strength, f'_c. The safety factor of 1.5, coupled with the overall requirement of a 1.2 safety factor on the charge weight, is required until there are adequate test data for spall and breach of masonry walls to determine a more accurate approach.

Additionally, CMU masonry with any ungrouted voids should not be used when the peak applied blast pressure exceeds 60 psi because of concerns that the face shell of the masonry may fail as a short beam spanning between webs at these pressures. This failure, which occurs during very early time response to the blast load, causes unreinforced walls to loose flexural capacity and may not allow ungrouted cells to span horizontally between reinforced cells as is typically assumed for reinforced walls that are not reinforced in each cell. This requirement restricts the

use of masonry walls with any ungrouted voids to far-range blast loading (i.e. scaled standoffs greater than 3 ft/lb$^{1/3}$).

Similarly, fragment penetration through masonry can be determined using the same approach shown in UFC 3-340-02 for concrete with several additional considerations. The thickness of ungrouted CMU or clay tile resisting penetration should only be taken as the minimum face shell thickness. Also, the masonry material compression strength, f'_{mc}, should be used in place of the concrete compression strength, f'_c, where f'_{mc} is the lesser of the masonry unit, mortar, or grout (if applicable) compression strengths. Calculations using this procedure overpredicted all measured penetration depths and matched perforation cases for experiments where twenty-five small steel cubes (i.e. 0.5 inch and 0.63 inch) were shot into unreinforced CMU, grouted CMU, and brick walls at velocities between 3000 ft/sec and 5000 ft/sec (reference).

Masonry Wall Construction

Masonry walls can be constructed as a single, monolithic wall with one or more units through the thickness acting compositely, or as cavity walls consisting of multiple, closely spaced masonry walls with an air gap or insulation between walls. New blast-resistant cavity walls should be designed assuming only the inner wall of cavity walls acts as a structural wall, and therefore requires reinforcement, with additional mass from the outer walls. The ties connecting the walls can be designed for conventional loads except that they need to be designed to transfer blast loads and rebound loads into the inner wall if failed debris from the outer wythe can be a hazard, including cases where the debris can fall onto an inhabited area or low roof area below.

Theoretically, cavity walls can be designed to act compositely, where the inner wall is in tension and outer wall is in compression. However, this approach has not been validated by test data and is not practical for new walls with steel reinforcement because of the large number of steel stirrups required between the wythes to transfer the full dynamic yield strength of the reinforcing steel into the adjacent wythe.

The foundation system can provide some level of rotational restraint at the bottom of masonry walls. Typically, the bottom of a masonry wall is connected with rebar dowels to a continuous concrete stem wall and spread footing. Conservatively, this connection can be analyzed as a pinned support, which will limit the wall's flexural resistance to blast load. This assumption also limits the reaction force at the foundation and the corresponding shear stress in the wall. To help ensure that the wall responds consistently with this assumption, the splice between the dowels and vertical reinforcing steel should be limited to the compression splice length as would be typical for conventional design. On the other hand, a full tension splice length can be provided and the wall can be designed with a fixed support at the foundation (i.e. no rotation) if it can be shown that the soil pressures based on ultimate soil bearing capacities acting on the stem wall and the footing can provide a resisting moment equal the ultimate dynamic moment capacity of the wall cross section at the base of the wall.

Figure 1 and Figure 2 illustrate the calculation of the resisting moment provided by the foundation at the bottom of a reinforced masonry wall using soil parameters that are intended to represent medium strength soils and a typical range of foundation dimensions. These calculated soil resisting moments can be compared for reference to an ultimate dynamic moment capacity of 11,400 lb-in/in from an 8 inch CMU wall reinforced at midthickness with a 5/8 inch rebar.

This comparison indicates that cohesive soils are more likely to provide the necessary moment resistance to cause fixity at the bottom of a reinforced masonry wall than granular soil.

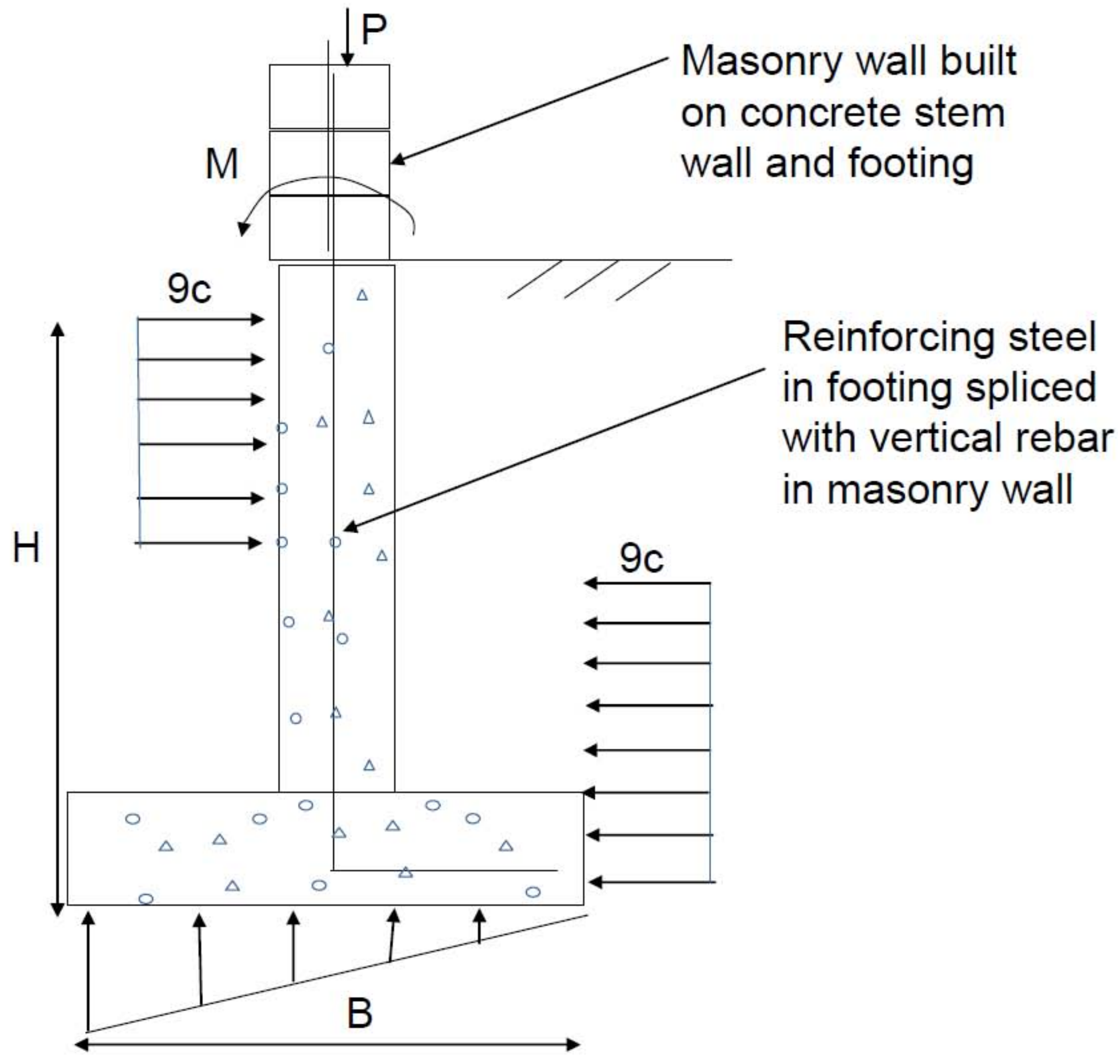
The resisting moment from cohesive soil in Figure 1 is calculated using Brom's method to determine lateral soil pressures against piles and footings. The resisting moment from granular soil in Figure 2 is calculated using Rankine's method to determine lateral soil pressures and provides significantly lower calculated resisting moments than the cohesive soil in the Figure 1. Other design-based methodologies commonly used for lateral design of foundation systems or finite-element methods validated against data may also be used.

Dynamic Analysis

The dynamic analysis of masonry walls can be modeled with an equivalent single-degree-of-freedom (SDOF) system, as explained in Chapter 3 of the UFC. Details for determining the resistance-deflection relationship are described in the following sections. The moment of inertia is equal to either the gross moment of inertia for unreinforced masonry, or to the average of the gross and fully cracked moment of inertia for reinforced masonry. At deflections greater than the yield deflection, the response of reinforced masonry walls may be considered ductile (i.e. it has a constant resistance equal to the ultimate flexural resistance with increasing deflection) while the response of unreinforced masonry walls is considered brittle and the flexural resistance goes to zero. However, the calculated response of unreinforced masonry walls can include the additional resistance at deflections greater than the yield deflection from the effects of in-plane axial force, including self-weight, which cause arching response. The deflections should be limited based on the response criteria for the desired protection category as shown in Table 3. These response criteria also apply during rebound.

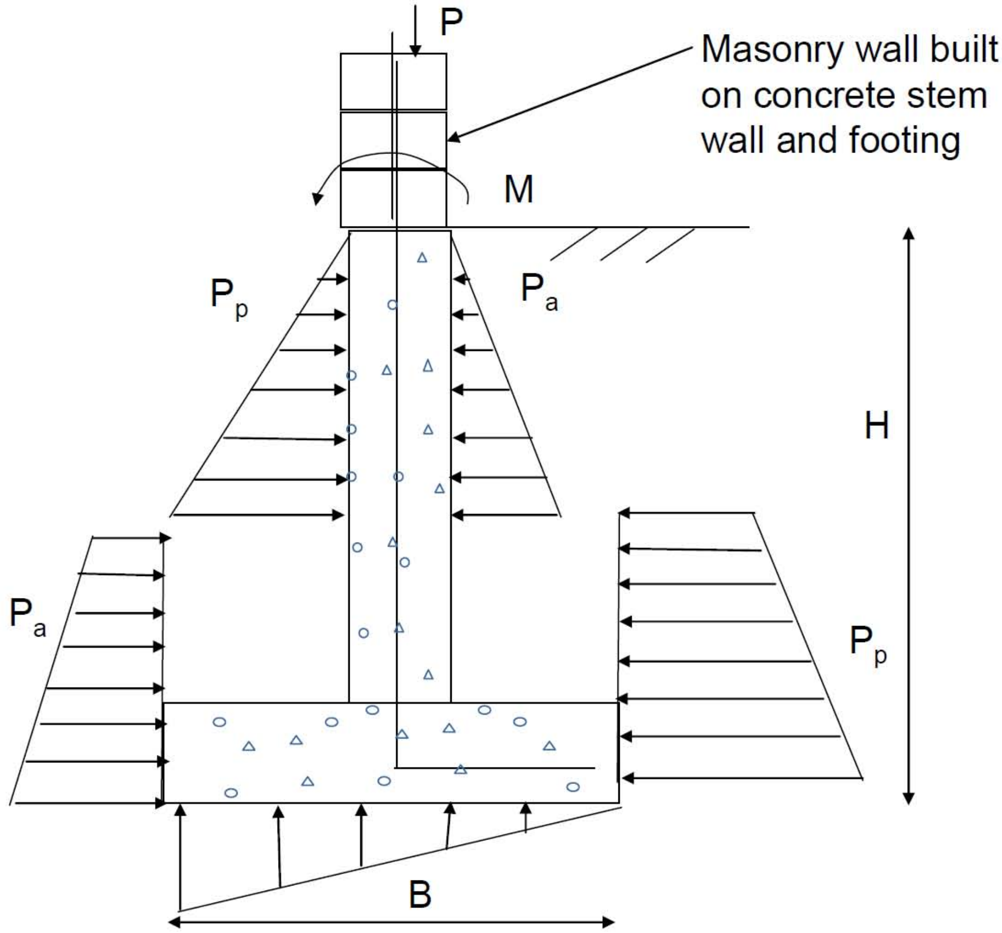
Table 3. Response Criteria for Masonry Blast Design

Wall Type	Support Rotation (Degrees)		Deflection to Thickness Ratio	
	Category 1	**Category 2**	**Category 1**	**Category 2**
Steel Reinforced Masonry	2.0	8.0	N/A	N/A
Unreinforced Masonry*	1.5	4.0	0.5	0.8
*Only applicable for brittle flexural response followed by compression membrane or axial load arching				



Height (H) (ft)	Cohesion (c) (psf)	Axial Load (P) (lb/in)	Width (B) (ft)	Resisting Moment lb-in/in
6	1000	35	5	64030
4	1000	35	4	28820
4	500	35	4	14810

Figure 1. Resisting Moment Provided to Foundation in Cohesive Soil



Height (H) (ft)	Angle of Friction (Deg.)	Coefficients of Soil Pressure		Soil Density (pcf)	Axial Load (P) (lb/in)	Width (B) (ft)	Resisting Moment lb-in/in
		Passive (K_p)	Active (K_a)				
6	30	3	0.33	95	35	5	6650
4	30	3	0.33	95	35	4	2400

Figure 2. Resisting Moment Provided to Foundation in Granular Soil

Reinforced Masonry Walls

Reinforced masonry walls are designed to resist blast load in flexure in a similar manner as reinforced concrete walls, except that the walls must comply with construction requirements and reinforcement steel detailing provisions of ACI 530. The applicability of this assumption to exterior masonry walls has been demonstrated in numerous static tests and blast tests on well constructed reinforced masonry walls (Oswald et al, 2006). Also, all blast resistant reinforced masonry walls should be subject to inspection and material testing during construction as required in ACI 530 for essential buildings. This requirement helps ensure that the reinforcing steel and surrounding grout will be constructed to act compositely with the surrounding masonry before and after yielding of the steel by preventing poor construction procedures such as insufficient vibrating during placement, improper rebar placement too near an edge of the grouted space, mortar protrusion and drippings into the grouted space, and grout with insufficient strength or plasticity.

The maximum bar size for primary vertical reinforcing steel is a #6 bar (0.75 inch diameter) to encourage the use of more distributed reinforcement, rather than the use of a smaller number of larger bars. This requirement also helps to limit the demand on the grout to provide composite action between the reinforcement and masonry through large plastic deflections. Minimum and maximum values for the amount of steel reinforcement are shown in Table 4. The minimum steel ratio for primary steel is based on the requirement in ACI 530 that the minimum steel ratio should cause a moment capacity that exceeds 1.3 times the masonry cracking moment. A dynamic modulus of rupture for masonry equal to 200 psi and dynamic reinforcing steel yield strength of 77 ksi are assumed. The maximum steel ratio is based on a conservative, historical value from the Unified Building Code of one-half the balanced steel ratio, assuming static prism strength of 2000 psi with a DIF of 1.2 and a dynamic yield strength of 77 ksi for the reinforcing steel. The minimum horizontal joint reinforcement is required to provide confinement around splices and distribute loads into the vertical reinforcing steel.

Table 4. Reinforcing Steel Limitation for Masonry

Case	Reinforcement Limitation
Minimum steel ratio for primary steel (vertical steel)	$A_s \geq 0.0006bd$
Maximum steel ratio for primary steel (vertical steel)	$A_s \leq 0.006bd$
Minimum secondary steel (horizontal joint reinforcement)	Two longitudinal W1.7 wires in bed joints at every other course (16 inches)

Reinforcement should have a minimum development length and splice length, l_d, as required in ACI 530 including the material reduction factor (i.e. φ factor). All splices should be located in regions of low stress (i.e. no more than 75% of the maximum moment) and should be specified on the structural drawings to help ensure that splices are placed as intended. Additionally, splices must be increased 30% if there are two bars per cell (four per cell at splice locations) or if any of the bars are within 3 inches of each other. CMU walls with one layer of spliced reinforcement (i.e. at mid-thickness) should have at least a nominal thickness of 8 inches to help provide sufficient grout in the cells around spliced reinforcing steel. CMU walls with spliced

reinforcement at each face should have at least a nominal thickness of 12 inches for the same reason.

Mechanical splices are not allowed until further research is conducted to determine their effectiveness at high strain-rates typical of blast loading. Welding of steel reinforcement is to be avoided for blast resistant design as discussed in the UFC for reinforced concrete. Reinforced CMU walls are typically built using low-lift construction with a maximum grout pour height of 5 ft with splices in vertical reinforcement at the top of each lift. The need to keep splices out of maximum moment regions may prevent the use of splices at the top of each grout pour and cause larger rebar length and associated block lift heights unless "A" blocks are used, which is permitted. Alternatively, high lift grout placement may be used according to the requirements of ACI 530, which allows much longer lengths of unspliced vertical reinforcement.

Unreinforced Masonry Walls

The flexural capacity of unreinforced masonry walls is based on a resisting moment from the dynamic tensile strength of the masonry, f_{dt}, as defined previously, and the elastic section modulus of the component. Additionally, the net tensile strength may include any precompression from supported dead load. After reaching this resistance, which typically occurs at very small deflections (i.e. tenths of an inch), the masonry will begin to fail in a brittle manner. There is assumed to be sufficient ductility to reach the ultimate resistance considering yield at all maximum moment locations of an indeterminate unreinforced masonry wall at the strain rates typical of blast response because of the very small additional deflections that are involved. Axial load arching occurs after brittle flexural response, which provides significantly more strain energy.

Figure 3 illustrated the mechanics of axial load arching. As significant rotation occurs at midspan and at the supports after brittle flexural failure, the axial load is resisted near the unloaded face near the supports and near the loaded face at midspan. The resulting in–plane axial force couple (i.e. axial load arching) causes a resistance equal to r_{al} in Equation 4. The peak magnitude of the resisting couple is a function of the applied axial load, including component self-weight above midspan, and component thickness. Note that the axial load from self weight in Equation 4 is assumed to act along the same line of action as the axial load, P, in Figure 3 when, in fact, it acts along a line of action through the center of gravity of the wall. This is a simplification that is supported by comparisons of SDOF analysis using Equation 4 to measured response of unreinforced masonry walls that generally did not have applied axial loads (i.e. only self weight) (PDC-TR 08-07, 2008).

$$r_{al} = \frac{8}{L^2}(h - x_3)\left(P + \frac{WL}{2}\right)$$

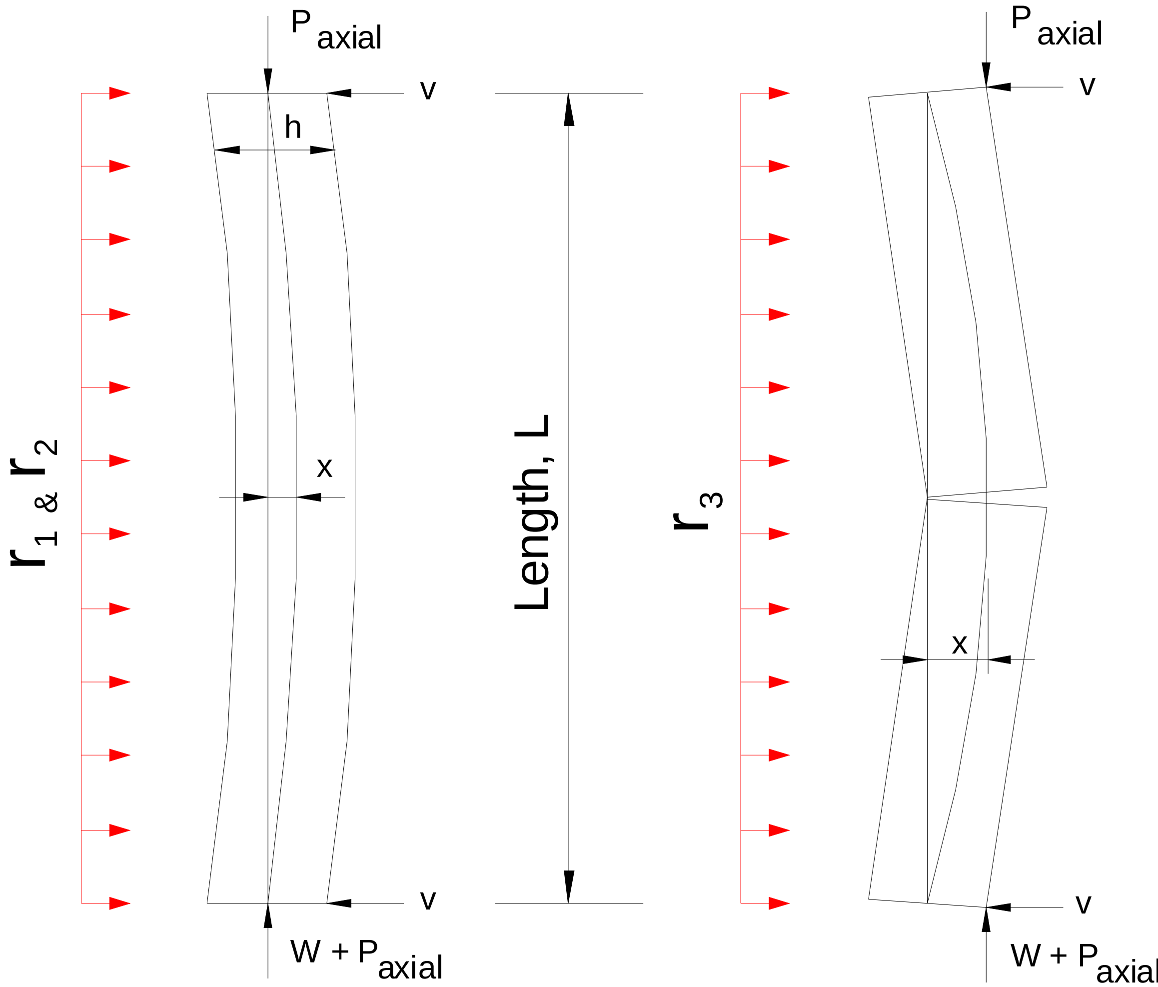
Equation 4

where:

r_{al} = maximum resistance from axial load effects
x_3 = deflection at beginning of axial load arching, see Figure 3. This can be assumed as the deflection at ultimate flexural resistance
h = overall wall thickness

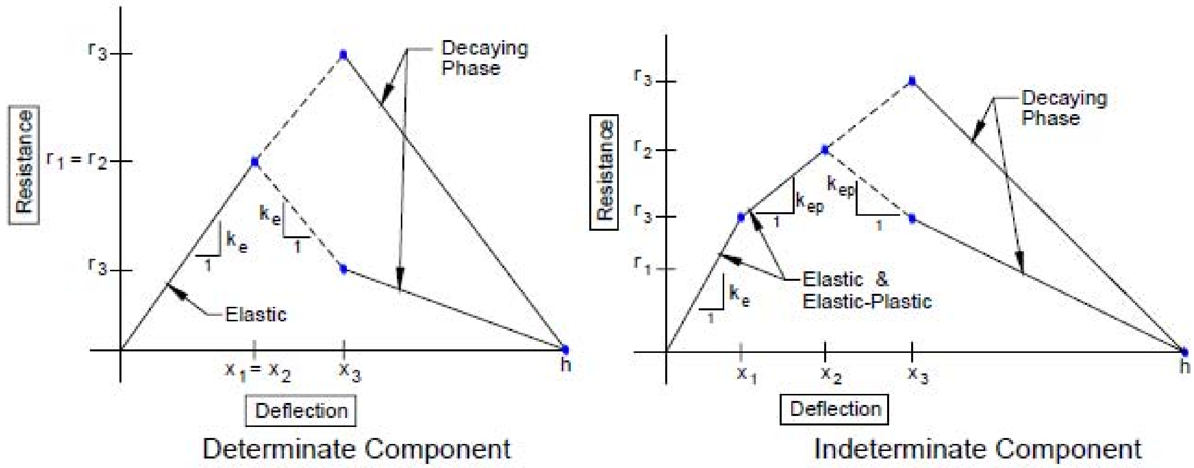
P = input axial load per unit width along wall, P_{axial}
W = areal self-weight and supported weight of wall
L = span length equal to wall height

Figure 4 depicts the proposed resistance-deflection relationships for unreinforced masonry with brittle flexural response and axial load arching. All resistance is due to flexural response until flexural yielding has occurred at all maximum moment regions and the component becomes a mechanism (i.e. at all deflections less than x_2 in Figure 4). At deflections between x_3 and the component thickness, h, (i.e. during the "Decaying Phase" of Figure 4) all resistance is due to axial load arching. The resistance-deflection relationship transitions between these two responses modes (i.e. between x_2 and x_3) in Figure 4 at an assumed slope equal to the elastic stiffness. For two-way spanning walls, axial load arching acts only in the vertical span direction.



Response prior to ultimate flexural resistance, r_2 Response after r_2 where x= arching mom

Figure 3. Axial Load Arching in Unreinforced Masonry Walls



Note 1: Upper curve applicable when $r_3 > r_2$
Note 2: Figures are not to scale since typically $x_3 << h$

Term in Figure	Definition
r_1	Initial flexural yield resistance
r_2	Ultimate flexural yield resistance
r_3	Maximum resistance from axial load arching (see Equation 6-10)
k_e	Initial flexural stiffness
k_{ep}	Flexural stiffness after initial yielding of indeterminate component
h	Wall thickness

Figure 4. Detailed Resistance-Deflection Curves for Unreinforced Masonry Wall with Axial Load Arching

Figure 5 shows a typical resistance-deflection curve for unreinforced masonry wall with axial-load arching. The peak axial load arching resistance, r_{al}, is very small in this case (i.e. 0.04 psi), but the strain energy from axial load arching is much greater than that from the flexural resistance that has a much higher peak resistance (i.e. 0.47 psi).

Unreinforced cavity walls can be designed as multiple walls responding together in flexure in a non-composite manner with equal deflections if it can be shown that the tie anchors will allow the outer wall to transfer load to the inner wall without failing. In this case, the system of walls has a total flexural stiffness equal to the summed flexural stiffnesses of all walls acting separately and it resists the blast load with stresses proportional to their relative flexural stiffnesses. The walls are assumed to resist lateral load in this manner until the wall with the highest stresses yields in tension at all maximum moment locations to become a mechanism. The combined flexural resistance of all the walls when this occurs is the ultimate brittle flexural resistance of the cavity wall system. Each wall must resist shear forces based on its individual flexural resistance at the deflection where the wall system becomes a mechanism. This

approach, which is analogous to a series of springs acting in parallel, is consistent with the approach recommended in ACI 530.

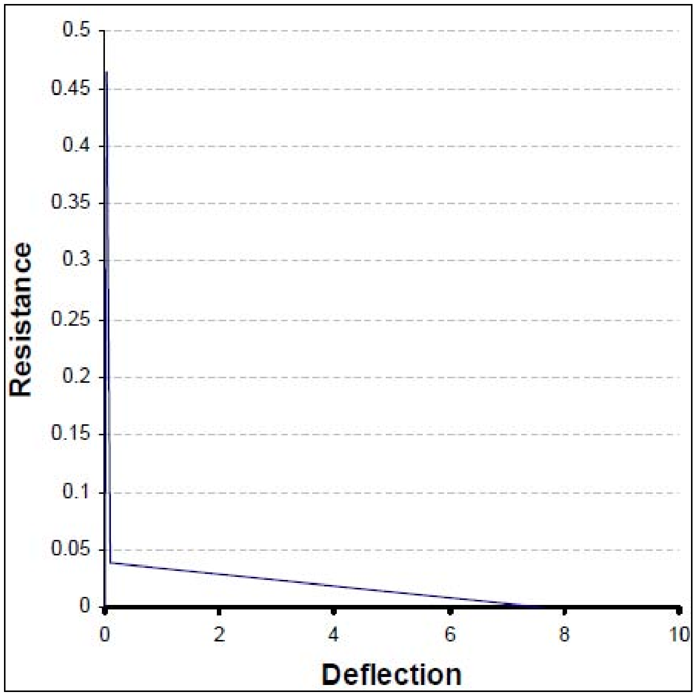


Figure 5. Resistance-Deflection Curve for Unreinforced Masonry Wall with Axial Load Arching

The response of a cavity wall system with adequate connectors will only increase significantly if the two wall thicknesses are nearly equal. For example, if the ratio of wall thicknesses in a two wall system is equal to 2.0, the value of ultimate flexural resistance of the two-wall system will only be 1.125 times greater than that for thicker wall. This contrasts sharply with the case where both walls are equal thickness, where the value of flexural resistance of the combined wall system is 2.0 times great than the resistance of either wall. Solid multi-wythe unreinforced masonry walls can act compositely if they are constructed with a solid grout fill and steel ties between the walls or masonry units from each wall protruding into the other wall (i.e. connecting headers) as required for composite multi-wythe wall construction in Chapter 2 of ACI 530.

Limits on the Use of Masonry Construction

The previous requirements and material strength limitations impose a number of limitations on the use of masonry for explosives safety applications that are summarized here. All new blast-resistant masonry construction must have vertical reinforcement. Non-solid masonry walls (i.e. CMU and clay tile) with any ungrouted voids should only be used to provide protection against far-range blast loading (i.e. only at scaled standoffs greater than 3 lb/ft$^{1/3}$). Solid reinforced masonry walls that provide required spall protection can be used at smaller scaled standoffs. Masonry cannot be used to resist internal blast loads except for the limited cases where the masonry is not subject to combined flexure and tension. For example, masonry walls may be designed to resist an internal blast load if they respond only vertically in one-way flexural response and are not subject to tension because the roof is lightweight material that fails quickly (i.e. vent roof). This includes the sidewalls and backwall of a test cell with a vent roof where the

roof framing provides adequate lateral support. Additionally, masonry walls should be designed to resist blast loads assuming they respond in flexure, with additional resistance from axial load arching in unreinforced masonry. The UFC does not allow design of any masonry walls with tension membrane and compression membrane response except for limited cases where DDESB approval is provided on a case-by-case basis.

Summary

PEC is working with DDESB to update the UFC 3-340-02, chapter 6 sections on blast resistant design of masonry components. The primary purpose of this revision is to provide updated and expanded guidance for the protective construction design of masonry walls to satisfy DoD 6055.09-STD explosives safety requirements. Wherever possible, the new UFC criteria will be written to be consistent with the current state-of-the-practice for conventional masonry construction. As part of this tasking, PEC is identifying areas where more research and testing is recommended. The UFC will continue to apply single-degree-of-freedom (SDOF) analytical modeling, but design procedures will be updated, in accordance with current industry practice, to use vertical steel reinforcing bars for flexural reinforcement. Flexural response criteria will be updated based on more recent blast tests. A new section will address analysis of existing unreinforced masonry components assuming brittle flexural response and arching from axial loads.

Updated guidance also will be provided on the calculation of compression strength, tensile strength, and shear strength of masonry; the rotational restraint provided to masonry walls by the foundation system; the calculation of the properties of the equivalent SDOF system for reinforced and unreinforced masonry walls; detailing of steel reinforcement; and inspection during construction. Finally, limitations on the use of masonry construction for explosive safety applications will be expanded and clearly stated.

PEC’s 35% draft revision currently is under review by TWG members. The projected completion date of the new masonry design sections is early FY 11. In the meantime, the authors welcome comments and suggestions from the explosives safety community.

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Update to UFC 3-340-02 for Blast Resistant Design of Masonry Components

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DDESB Explosive Safety Seminar
July, 2010

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Overview

- UFC 3-340-02 “Structures to Resist the Effects of Accidental Explosives” is being updated
  - Converted to UFC from previous title of “TM 5-1300” and into more accessible electronic format
  - Revisions to Chapter 4 on reinforced concrete
- Current task to update the masonry section in Chapter 6
- More updates will follow as funding becomes available

Updated Masonry Section

- Continues to use single-degree-of-freedom (SDOF) analytical models for design
- Reinforced masonry based on vertical steel reinforcing bars for flexural reinforcement
- New guidance on the analysis of existing unreinforced masonry components and for masonry shear strength
- New response criteria for each protection level
- Addresses spall, breach, and fragment penetration

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Updated Masonry Section (Cont'd)

- References to applicable ACI 530 and ACI 318 requirements
- Addresses rotational restraint provided to masonry walls by the foundation system
- Consistent with the current state-of-the-practice for conventional masonry construction whenever possible

Status of Project to Update Masonry Section

- Approach was presented at initial meeting with UFC 3-340-02 Working Group with feedback
- 35% submittal has been completed and reviewed by Working Group
- National Concrete Masonry Association (NCMA) review underway
- Update should be completed by end of year
- Comments are welcomed

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Masonry Material Properties

- Compression strength based on masonry prism strength f'_m
- Dynamic increase factors for flexural response
- Dynamic tensile strength of unreinf. masonry
  - Between 200 psi and 250 psi based on f'_m
  - Based on matching SDOF analysis with test results
- Shear strength based on f'_m and net cross sectional area
  - In-plane and out-of-plane shear
  - Shear strength can include contribution from steel reinforcement
  - Stirrups not practical for out-of-plane shear

Default Prism Strengths and Dynamic Increase Factors

Type of Unit	Prism Strength (f'_m)
Hollow Units	1350 psi
Hollow Units Filled with Grout	1500 psi
Solid Units	1800 psi

Material	Response Mode	Dynamic Strength
Masonry	Compression due to Flexure	$1.19\, f'_m$
	Shear	$1.00\, f'_m$
	Axial Compression	$1.12\, f'_m$
Steel Reinforcement	Tension due to Flexure	$1.17 f_y$(SIF)

f'_m is masonry prism strength
f_y is minimum specified reinforcing steel yield strength
SIF is the static increase factor equal to 1.1

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Proposed Masonry Shear Strength

- Shear strength based on net shear area
- Zero shear strength if full thickness tension
  - Typical case for walls resisting internal explosion

$$V_m = 2\sqrt{f'_m}A_n$$

V_m = dynamic shear strength per unit width along wall resisted by masonry (lb)
f'_m = masonry prism compressive strength (psi)
A_n = net cross section area for shear (in^2)
= solid cross section (i.e. exclusive of any void area) for out-of-plane shear
= area of face shells of non-solid masonry for in-plane shear
= whole thickness in all cases for solid masonry

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Combined Requirement for In-Plane and Out-of-Plane Shear Loads

$$\left(\frac{V_{uo}}{V_{mo}+V_{so}}\right)^2+\left(\frac{V_{ui}}{V_{mi}+V_{si}}\right)^2 \leq 1.0$$

V_{mo} = out-of-plane shear force resisted by masonry (lb)
V_{so} = out-of-plane shear force resisted by shear reinforcement (lb)
V_{mi} = in-plane shear force resisted by masonry (lb)
V_{si} = in-plane shear force resisted by shear reinforcement (lb)
V_{uo} = peak applied out-of-plane shear force (lb)
V_{ui} = peak applied in-plane shear force (lb)

Spall, Breach, Fragment Penetration

- Spall and breach based on Chapter 4 approach for reinforced concrete (RC)
  - Only applicable for solid masonry
  - Required thickness to prevent spall and breach for RC increased by 1.5 safety factor
- No use of ungrouted CMU at peak applied pressures greater than 60 psi
  - Prevent failure of face shells between webs
- Fragment penetration based on Chapter 4 approach for reinforced concrete (RC)
  - Use least masonry material compression strength f'_{mc} in place of the f'_c for RC
  - Compares conservatively to test data on CMU and brick

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Masonry Wall Construction

- Walls can be constructed as single monolithic wall or as cavity walls
  - Single wall can have one or more units through the thickness acting compositely
  - Cavity walls are separate closely spaced walls connected by ties
- Typically outer wall of cavity wall is non-structural (only contributes mass)
  - Inner wall must be reinforced for new walls
  - No special requirements for ties if non-structural outer wall, unless wall debris is hazardous

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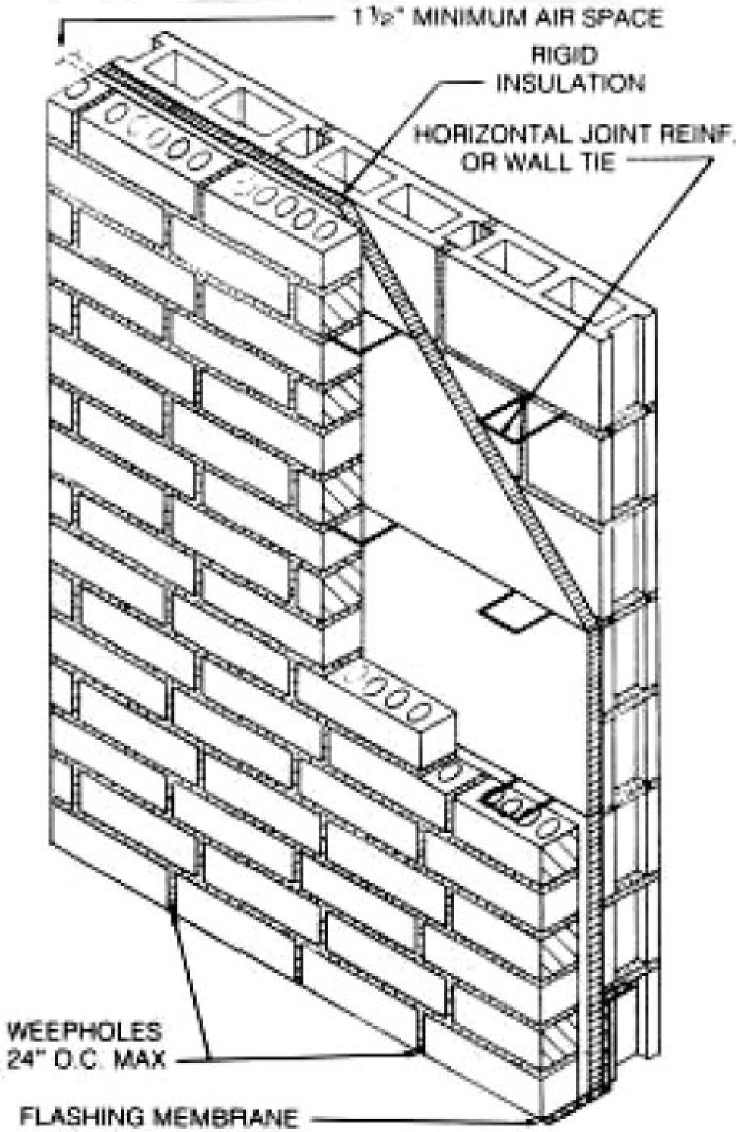
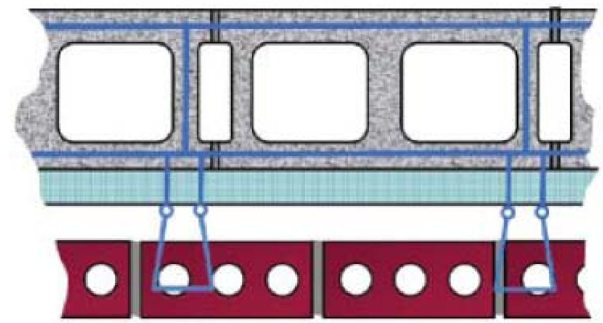
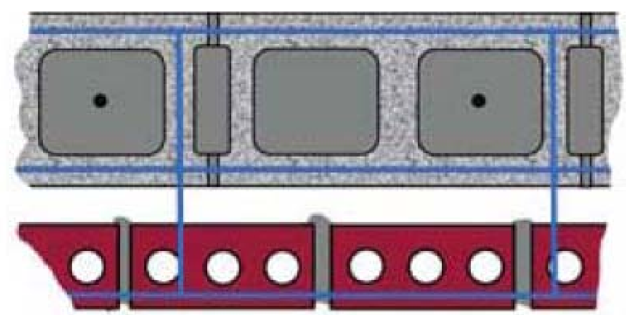
Typical Cavity Wall Construction





Unreinforced wall with adjustable tie anchors



Reinforced wall with fixed tie anchors

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Restraint by Foundation

- The foundation system can provide rotational restraint at the bottom of masonry walls
- Typically, bottom of a masonry wall connected to foundation with rebar dowels
- Connection may be simple support for reinforced wall if compression only splice between dowels and vertical reinforcement
  - Limits wall's flexural resistance to blast load and corresponding shear stress in the wall.
- A full tension splice can cause a fixed support if soil pressures can provide resisting moment equal to that in reinforced wall
  - Based on ultimate soil bearing capacities acting on the foundation stem wall and the footing
- Dowels cause fixity in unreinforced wall

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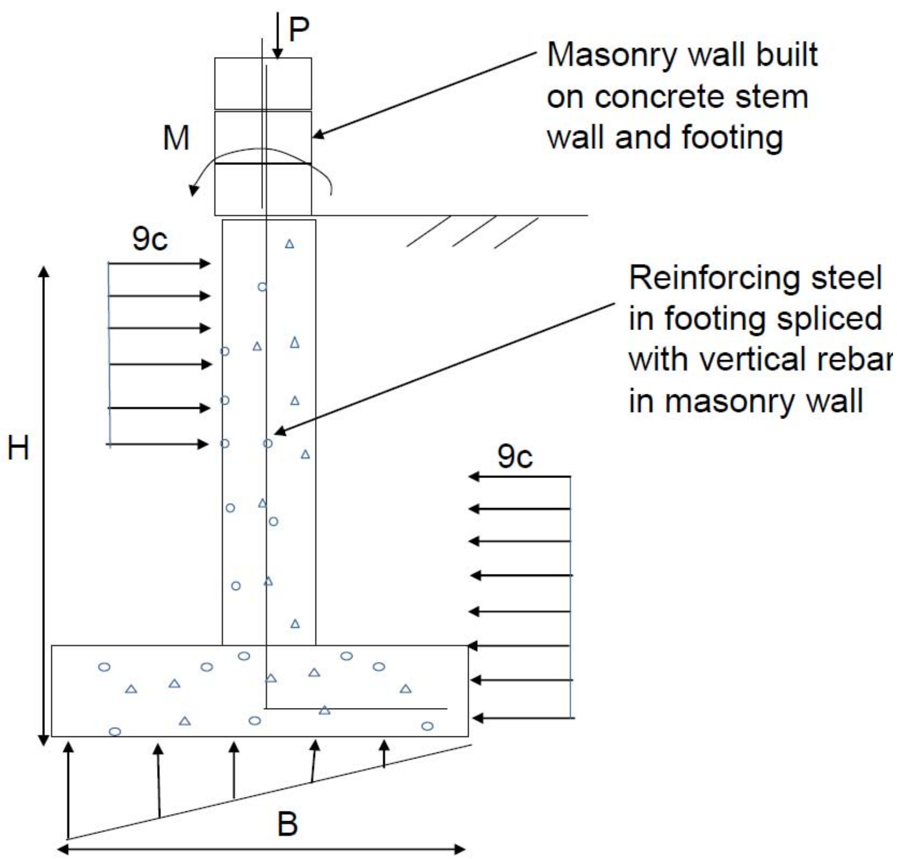
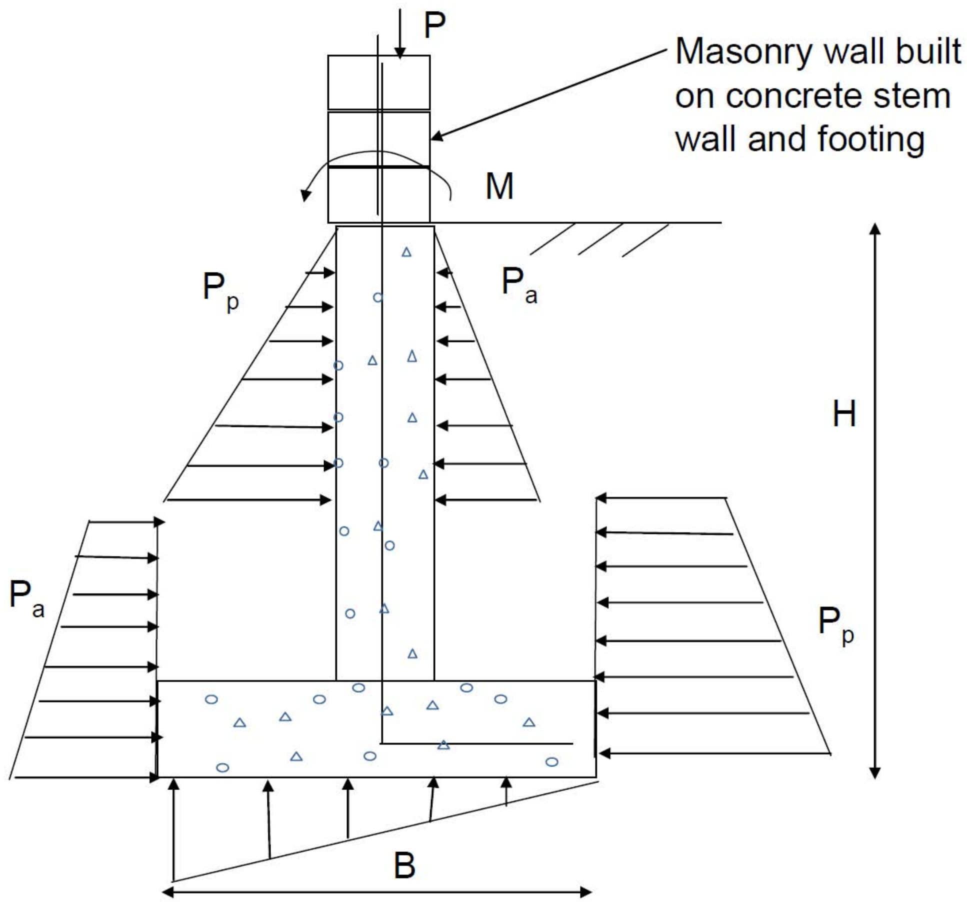
Moment Restraint Provided by Foundation



Clay Soil (Brom's Method)



Granular Soil (Rankine's Method)

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Moment Restraint Provided by Foundation

Clay Soil

Height (H) (ft)	Cohesion (c) (psf)	Axial Load (P) (lb/in)	Width (B) (ft)	Resisting Moment lb-in/in
6	1000	35	5	64030
4	1000	35	4	28820
4	500	35	4	14810

Granular Soil

Height (H) (ft)	Angle of Friction (Deg.)	Coefficients of Soil Pressure		Soil Density (pcf)	Axial Load (P) (lb/in)	Width (B) (ft)	Resisting Moment lb-in/in
		Passive (K_p)	Active (K_a)				
6	30	3	0.33	95	35	5	6650
4	30	3	0.33	95	35	4	2400

Note: Ultimate dynamic moment capacity of a 8 inch CMU wall #5 bar at 12” inch is 11,400 lb-in/in.

Reinforced Masonry Walls

- Walls analyzed as equivalent SDOF systems in flexure
  - Similar to approach in Chapter 4 for reinforced concrete with plastic yielding
- Requirements of ACI 530 for lap lengths, rebar placement, construction inspection apply
  - Inspection criteria as for “essential building”
- No splices in maximum moment region
  - Show splice locations on plans

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Proposed Steel Reinforcement Limits

- Minimum steel ratio causes a moment capacity equal to 1.3 times the masonry cracking moment (ACI-530)
- Maximum steel ratio is based on a conservative, historical value from the Unified Building Code of one-half the balanced steel ratio

Case	Reinforcement Limitation
Minimum steel ratio for primary steel (vertical steel)	$A_s \geq 0.0006bd$
Maximum steel ratio for primary steel (vertical steel)	$A_s \leq 0.006bd$
Minimum secondary steel (horizontal joint reinforcement)	Two longitudinal W1.7 wires in bed joints at every other course (16 inches)

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Unreinforced Masonry

- Existing walls may be analyzed to determine protection level against calculated blast load
- Resistance-deflection curve based on brittle flexural response followed by axial load arching
- Dynamic masonry tensile strength used for flexural resistance
- Unreinforced cavity walls can be analyzed as separate walls deflecting together until one wall fails in flexure

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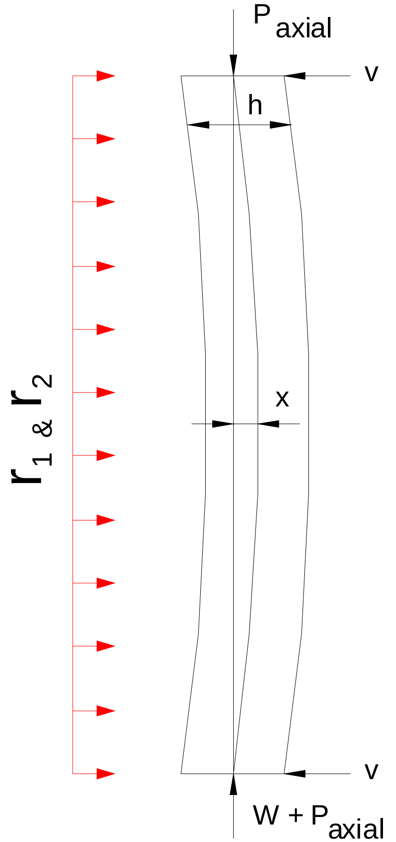
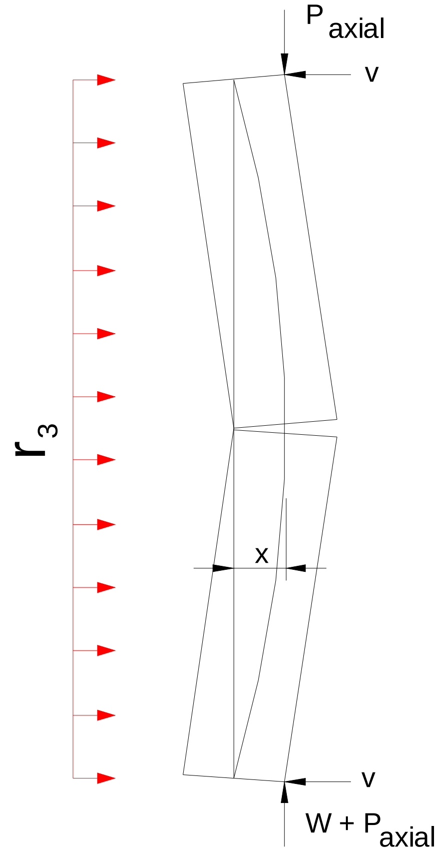
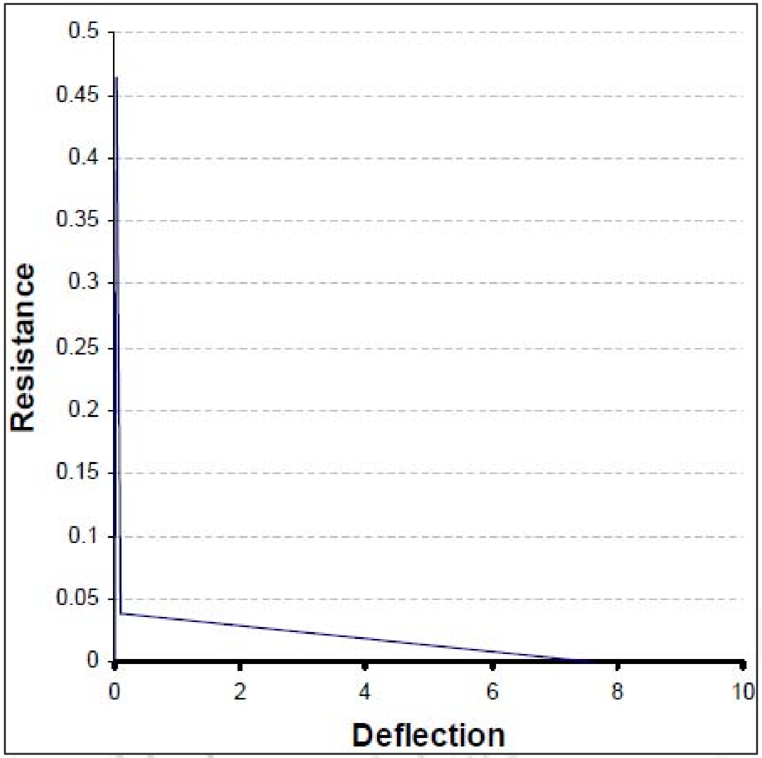
Resistance-Deflection Curve for Unreinforced Masonry



Brittle Flexure



Axial Load Arching



Typical Resistance-Deflection Curve

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Proposed New Response Limits

- New response limits based on more recent blast test data on reinforced and unreinforced masonry walls

Wall Type	Support Rotation (Degrees)		Deflection to Thickness Ratio	
	Category 1	Category 2	Category 1	Category 2
Steel Reinforced Masonry	2.0	8.0	N/A	N/A
Unreinforced Masonry*	1.5	4.0	0.5	0.8

*Only applicable for brittle flexural response followed by compression membrane or axial load arching

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Limitations on Use of Masonry Construction

- All new masonry construction shall have a minimum level of vertical steel reinforcement
- Non-solid masonry walls (i.e. CMU and clay tile) with any ungrouted voids only used for far-range blast loading ($Z > 3$ ft/lb$^{1/3}$)
- Masonry can only be designed to resist internal blast loads when not subject to combined flexure and tension
- Walls designed to respond in flexure, with additional resistance from axial load arching in unreinforced masonry.
  - No design with tension and compression membrane response except for limited cases with specific DDESB approval

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